Adobe Creek Reaches 5 & 6

Hydrology Report

Prepared by

James Wang, PhD, PE
Engineering Unit Manager

Wendy Chang, PE
Associate Civil Engineer

Nahm Lee
Assistant Engineer II

Santa Clara Valley Water District

February, 2005

GB
1227
A36.W3
2005
c.1
LIBRARY

SANTA CLARA VALLEY WATER DISTRICT
LIBRARY
5750 ALMADEN EXPRESSWAY
SAN JOSE, CALIFORNIA 95118

GB1227A36W32005c.1

TABLE OF CONTENTS

1. INTRODUCTION 1
   - 1.1 Purpose of the Study 1
   - 1.2 Flood Control Hydrology 1

2. PREVIOUS STUDY 1

3. UPDATED HYDROLOGY PROCEDURE 2

4. HYDROLOGIC MODEL 2
   - 4.1 Sub-Basin Boundaries 2
   - 4.2 Design Storm Precipitation 3

5. ROBLEDA TRIBUTARY 3

6. DESIGN FLOOD FLOWS 4

7. FLOOD FREQUENCY INFORMATION 5

REFERENCES 6

APPENDICES
- Appendix A Hydrologic Procedures
- Appendix B Hydrologic Parameters

TABLES
- Table 1 Flow Rates at Selected Catch Points
- Table 2 Flood Frequency at Various Locations

FIGURES
- Figure 1 Adobe Creek Reaches 5&6 Contour Map
- Figure 2 Adobe Creek Reaches 5&6 Watershed Sub-Basin Boundaries
- Figure 3 Adobe Creek Reaches 5&6 Parcel Map with Watershed Sub-basin Boundaries
- Figure 4 Adobe Creek Reaches 5&6 Detailed Hydrology for 1% 1(100-yr) Event
- Figure 5 Adobe Creek Reaches 5&6 Flood Frequency at Various Locations

1. INTRODUCTION

Adobe Creek originates in the northeasterly slopes of the Santa Cruz Mountains near Monte Bello Ridge. It drains 10 square miles and flows 14.7 miles from the Town of Los Altos Hills through the cities of Los Altos, Mountain View and Palo Alto before combining with Barron Creek. The Adobe-Barron Creek flow then enters the Palo Alto Flood Basin and subsequently flows into San Francisco Bay.

Adobe Creek Reach 5 extends from downstream of Foothill Expressway to downstream of West Edith Avenue for a total length of 2,270 feet, while Reach 6 extends from downstream of West Edith Avenue to downstream of Burke Road for a total distance of 2,335 feet.

1.1 Purpose of the Study

The main objective for this hydrology study is to develop the one percent and ten percent flow rates for Adobe Creek Reaches 5 & 6 using the District's updated hydrology procedures. The following catch points along Adobe Creek Reaches 5 & 6 were selected to calculate the one percent and ten percent peak flows for subsequent hydraulic analyses using a two-dimensional flow model:

- Adobe Creek at Burke Road
- Adobe Creek at Robinson's Property
- Adobe Creek at West Edith Ave. Bridge
- Adobe Creek at Astiz's Property
- Adobe Creek at Lippe's Property
- Adobe Creek at Foothill Expressway

1.2 Flood Control Hydrology

Hydrology is the study of the natural water cycle. The cycle begins with evaporation of water from the oceans and other bodies of water, continues with the formation of clouds and precipitation (rainfall), then transpiration through plants and their leaves, infiltration into the ground, runoff into streams, creeks and channels to the oceans, where the cycle begins again.

Flood protection hydrology mainly deals with the estimation of flood flows that can be generated from a watershed for a given frequency of occurrence. For example, in order to design a flood protection channel, engineers need to know the peak flow rate, which is the maximum rate of runoff, in the creek. The peak flow rate can be analyzed for any frequency of occurrence; the nationwide standard for flood protection design is the 100-year or 1% flood. A 1% flood has a 1/100th chance of being equaled to or exceeded in any one year.

2. PREVIOUS STUDY

In July of 1978, the Santa Clara Valley Water District (District) completed a study of the design flood flows which would be utilized in designing flood protection facilities within Santa Clara County. The study included the calculation of 1% and 10% design flood flows following the District's established method and procedures at the time.

The method in determining the design flood peaks and volumes utilized all available sources of information. It began with the use of Regional Regression Equations to compute peaks and volumes for rural areas. The values were also compared with the results obtained from a hydrologic rainfall-runoff model which is the HEC-1 Flood Hydrograph Package developed by the U.S. Army Corps of Engineers. Hydrographs were obtained from the model applying a 1% rainfall amount derived statistically from data at the nearby rainfall stations with the use of U. S. Army Corps of Engineers' 24-Hour Standard Project General Storm Precipitation Distribution for Northern California and the Clark Synthetic Unit Hydrograph technique. The peaks and volumes of these hydrographs were calibrated to be in close agreement with flood flow quantities derived from regional statistics. The flood hydrographs obtained this way were considered representative for use in the design of the flood protection facilities.

Based on the1978 hydrology study, the 1% flow rate in Adobe Creek at Foothill Expressway was about 2,700 cfs and the 10% flow rate was about 1,400 cfs.

3. UPDATED HYDROLOGY PROCEDURE

After the adoption of the District's hydrology procedures in the late 1970's, the USGS Office of Water Data Coordination changed its procedures in determining the flood flow frequencies of gaged data, which had been the basis for the development of the District's 1976 Regional Regression Equation. By the year 2000, with an additional 25 years of precipitation and stream flow data having been collected, the District believed it would be prudent to update the hydrology for Santa Clara County to keep up with current practices.

To update the District's hydrology procedures, comprehensive precipitation and stream flow data compilations and analyses were conducted. A new equation was developed to estimate the rainfall amount for those areas without rainfall gages. Similarly, with more historical stream flow data becoming available in Santa Clara County, a set of updated Regional Regression Equations was developed. The Regional Regression Equations for precipitation and streamflows are listed in Sections 1 and 2 of Appendix A.

4. HYDROLOGIC MODEL

The hydrologic model used for this study is the latest HEC-1 Flood Hydrograph Package (June, 1998). Details of the procedures described in the following sections can also be found in the District's publication entitled *Hydrology Procedures* (July 2004). The pertinent hydrologic modeling parameters and procedures in determining the design flow hydrographs are included in Appendix A.

4.1 Sub-Basin Boundaries

In addition to field observation, the following maps were used to delineate the watershed sub-basin boundaries.

1. Permanente Creek Flood Plain Map (June, 2004), Sheet 19 on Adobe Creek, developed by Aelytek Inc.. This map, with 0.5m contour lines asshown in Figure 1, was provided by the SCVWD Survey and Mapping Unit, and has been converted to a DEM (Digital Elevation Model) file.
2. Storm Drain Map from "DRAFT Town of Los Altos Hills Storm Drainage Master Plan (Oct, 2004)".
3. Santa Clara County Ortho Photos (2001)

The watershed sub-basin boundary delineation assumes that storm drains less than 24 inches in size will be full during both the 1% and 10% flood event, and the flow of water will follow the contour of the land. The watershed sub-basin boundaries are shown in Figures 2 and 3 with picture inserts showing certain hydrologic features.

Field observations indicate the following:

1. Sub-basin IR11 may not flow into sub-basin IR06 due to a barrier wall built by the resident. Thus, flow may follow the road draining to sub-basin IR10.
2. The flow direction for sub-basin IR14 will follow Fremont Road to sub-basin IR16, due to an existing retaining wall.
3. Sub-basin IR15 may flow over West Edith Avenue Bridge to sub-basin IR16, and then into Adobe Creek.

4.2 Design Storm Precipitation

Using the District's updated hydrology, the design storm precipitation has been developed and used to estimate the flood runoff magnitude. For this study, precipitation was obtained by data transpositioned from gaged locations to ungaged site and checked by using the global regional equation (Appendix A, Section 1.1) for reasonableness. For this study, the District's precipitation gages at Dahl Ranch (station #24), Maryknoll (station #53) and Mt. View Corp. Yard (station #121) were used for the data transposition. The resultant rainfall is 5.42 inches for a 24-hour, 1% event, and 3.98 inches for a 24-hour, 10% event.

5. ROBLEDA TRIBUTARY

The Robleda storm drain is a 60-inch diameter concrete pipe which was constructed in 1998 to relieve some of the local flooding. Its alignment is shown in Figure 2. This study assumes that the Robleda Tributary will carry all of the flows from sub-basin IR08 through IR14.

6. DESIGN FLOOD FLOWS

The design flows for this Adobe Creek Reaches 5 & 6 hydrology study are based on the 2004 updated hydrology procedure. After applying the hydrology procedure with precipitation, loss rates and routing parameters, the resulting flow rates are lower than the values obtained in 1978. Table 1 is a summary of the 1% and 10% flood flow rates at various catch points along Adobe Creek Reaches 5 and 6. The updated 1% flow rate at Foothill Expressway is 2,571 cfs and the 10% flow rate is 1,333 cfs. Figure 4 depicts the location of these calculated flow rates. Some detailed information on the parameters used for this study is included in Appendix B.

Table 1. Flow Rates at Selected Catch Points.

Catch Points	Basin Area	100-Year Peak Flow rate	10-Year Peak Flow rate
	square miles	cfs	cfs
Adobe Creek @ Burke Rd	6.48	2,390	1,243
Adobe Creek @ Robinson's Property	6.50	2,391	1,243
Adobe Creek @ Edith Ave. Bridge	6.53	2,397	1,245
Peak Flow into Robleda Trib.(IR08-12)	0.36	118	64
Peak Flow into Robleda Trib.(IR13 & 14)	0.02	7	4
Overland flow along Fremont Rd (IR05-07)	0.15	47	25
Overland flow above W. Edith Ave. Bridge(IR15)	2.8 Acres	2	1
Overland flow at the Park & Astiz's property (IR16)	3.3 Acres	2	1
Total overland flow at Astiz's property (IR15 & 16)	6.1 Acres	4	2
Adobe Creek @ Astiz's property	6.69	2,446	1,270
Adobe Creek @ Lippe's property	7.08	2,559	1,330
Adobe Creek @ Foothill Expressway	7.13	2,571	1,333

7. FLOOD FREQUENCY INFORMATION

The flood frequencies for Adobe Creek at various locations are listed in Table 2 with graphical presentation shown in Figure 5. The 1% and 10% peak flow rates are from HEC-1 rainfall-runoff modeling, while the remaining flows are interpolated and extrapolated by assuming a log-Pearson Type III distribution for various probabilities. These flow rates can be used for subsequent hydraulic analyses to better understand how flow behaves over the study reach.

Table 2. Flood Frequency at Various Locations

Probability (%)	Return Period (years)	Peak (cfs)			
		At Burke Road	At Edith Ave. Bridge	At Lippe's Property	At Foothill Expressway
0.2	500	3,160	3,170	3,380	3,400
0.5	200	2,730	2,730	2,920	2,930
1	100	2,390	2,397	2,559	2,571
2	50	2,050	2,060	2,190	2,200
4	25	1,700	1,710	1,820	1,830
5	20	1,590	1,600	1,700	1,710
10	10	1,243	1,245	1,330	1,333
20	5	895	895	955	960
40	2.5	545	545	580	580
43	2.33	510	510	540	540
50	2	430	430	460	460
66	1.5	280	280	300	300
90	1.1	110	110	115	115
99	1	30	30	30	30

REFERENCES

ADS Consultants, Saah, A. D. et el., *Development of Regional Regression Equations to Calculate Flood Quantiles in Santa Clara County*, San Jose, CA: Santa Clara Valley Water District, October, 2003.

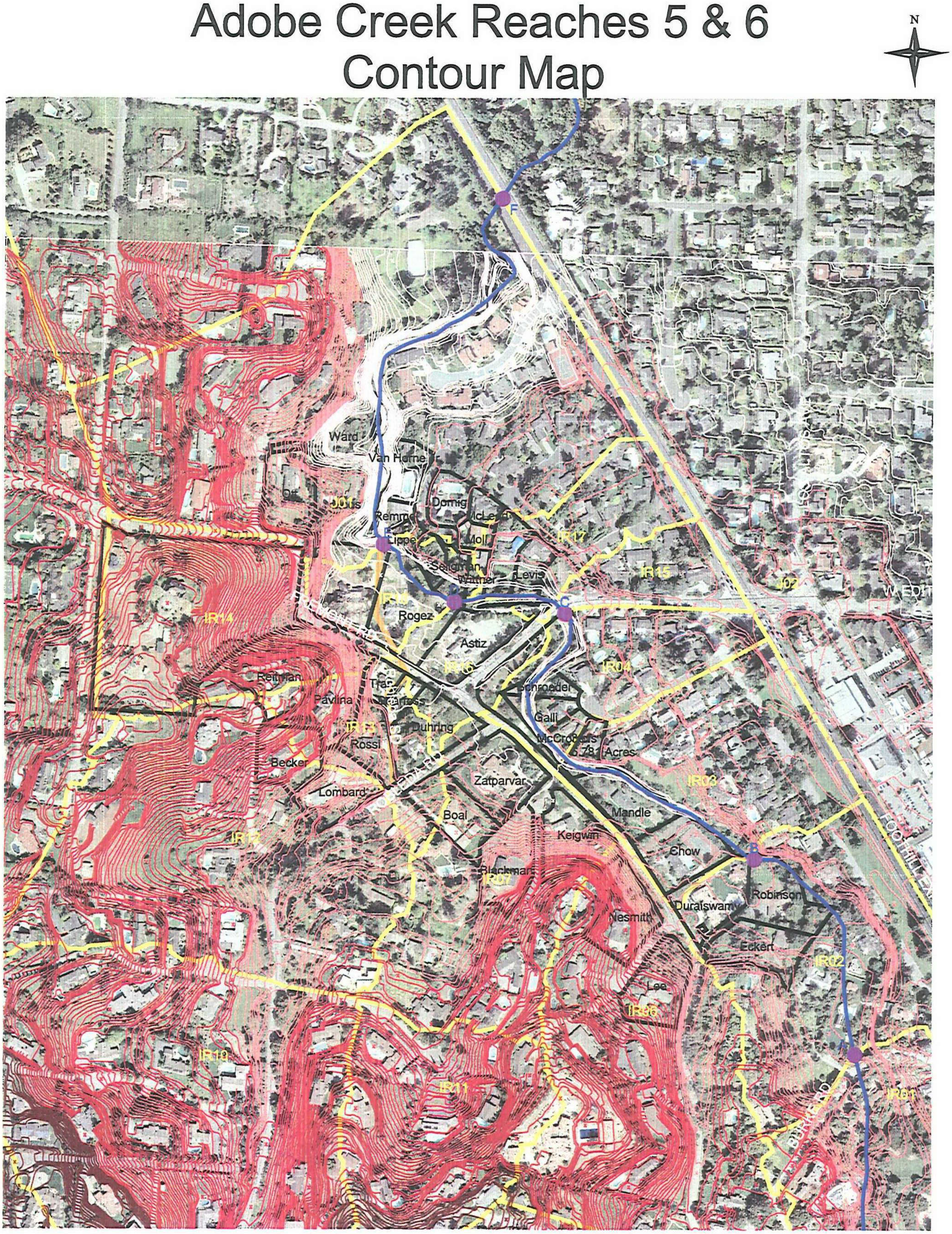
Flood Control Planning Unit, *Adobe Creek Watershed Planning Study*, San Jose, CA: Santa Clara Valley Water District, October, 2003.

Saah, A., G. Q. Tabios III, W.W. Chang and B.S. Jaber, *Precipitation-Duration-Frequency analysis and Modeling for Santa Clara County and Vicinity*, San Jose, CA: Santa Clara Valley Water District, October, 1996.

Wang J., W. W. Chang and Lee N., *Hydrology Procedures*, San Jose, CA: Santa Clara Valley Water District, Draft Report, July, 2004.

Wang J., W. W. Chang and Lee N., *Lower Peninsula Watershed Hydrology Report*, San Jose, CA: Santa Clara Valley Water District, Draft Report, November, 2003.

Adobe Creek Reaches 5 & 6
Contour Map



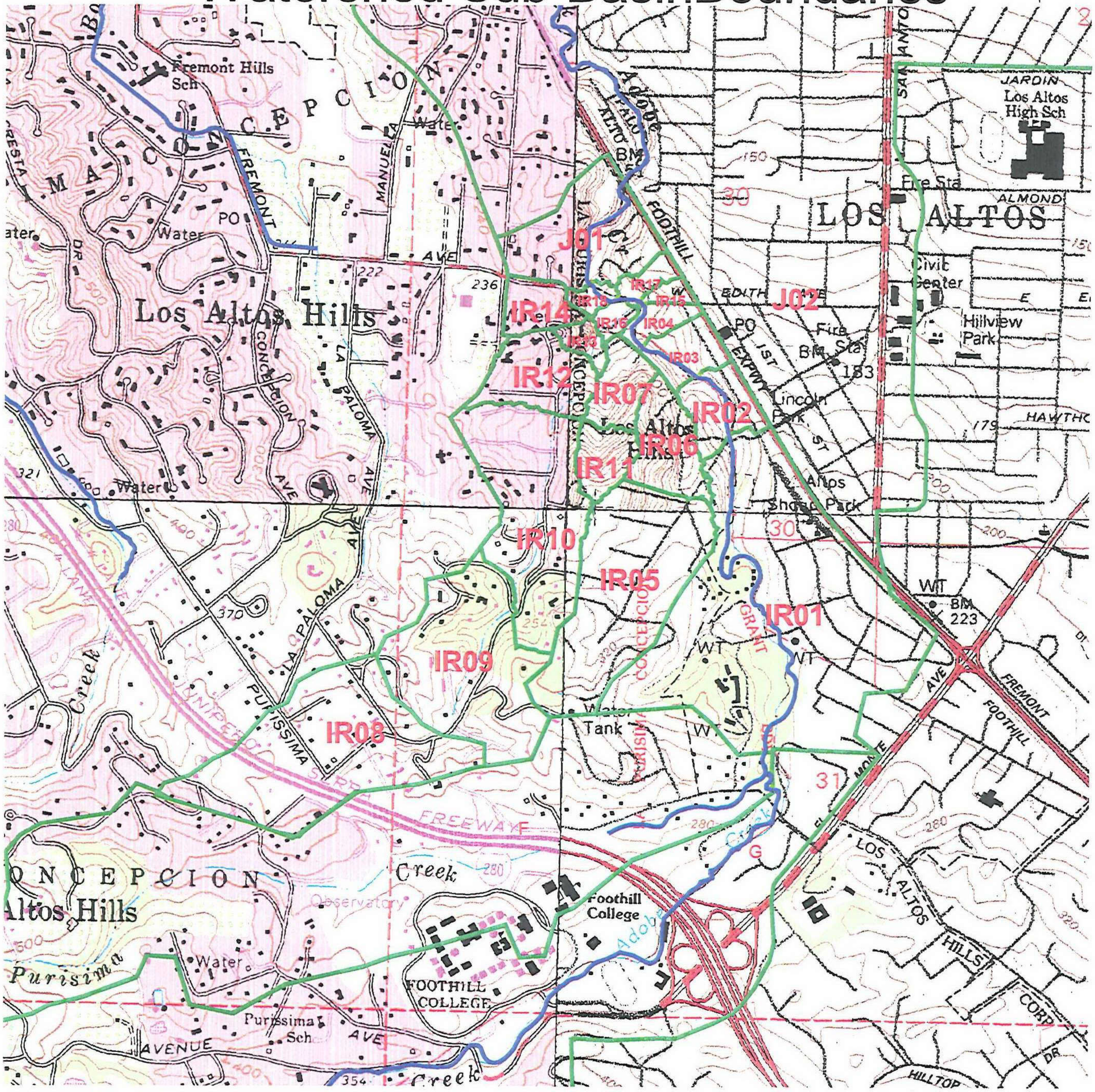
200 0 200 400 Feet

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

February, 2005

Figure 1. Adobe Creek Reaches 5 & 6 Contour Map

Adobe Creek Reaches 5 & 6
Watershed Sub-BasinBoundaries



800 0 800 1600 Feet

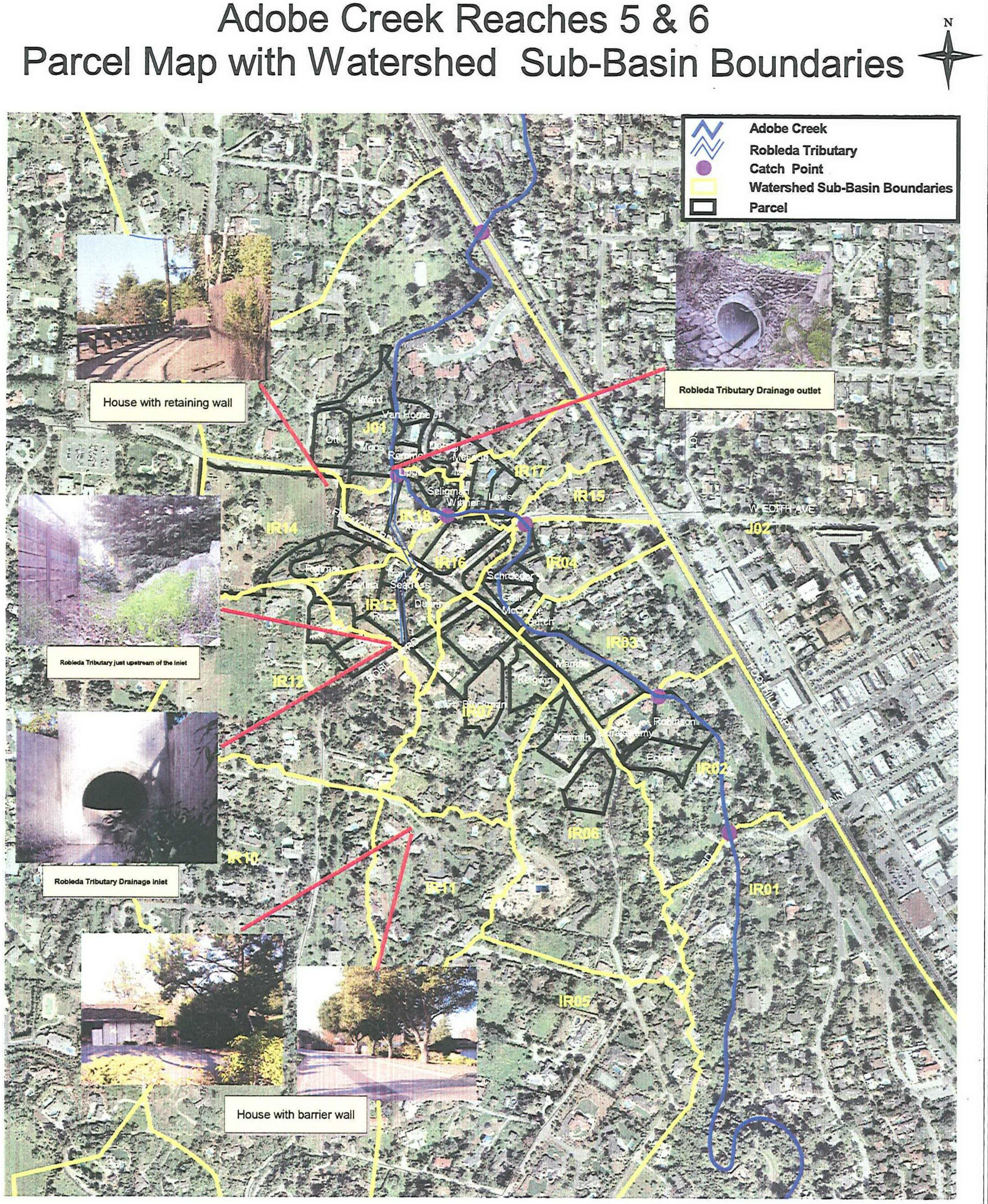
Creek
Watershed Sub-Basin Boundaries

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

February, 2005

Figure 2. Adobe Creek Reaches 5 & 6 Watershed Sub-Basin Boundaries

Adobe Creek Reaches 5 & 6
Parcel Map with Watershed Sub-Basin Boundaries



Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

February, 2005

Figure 3. Adobe Creek Reaches 5 & 6 Parcel Map with Watershed Sub-Basin Boundaries

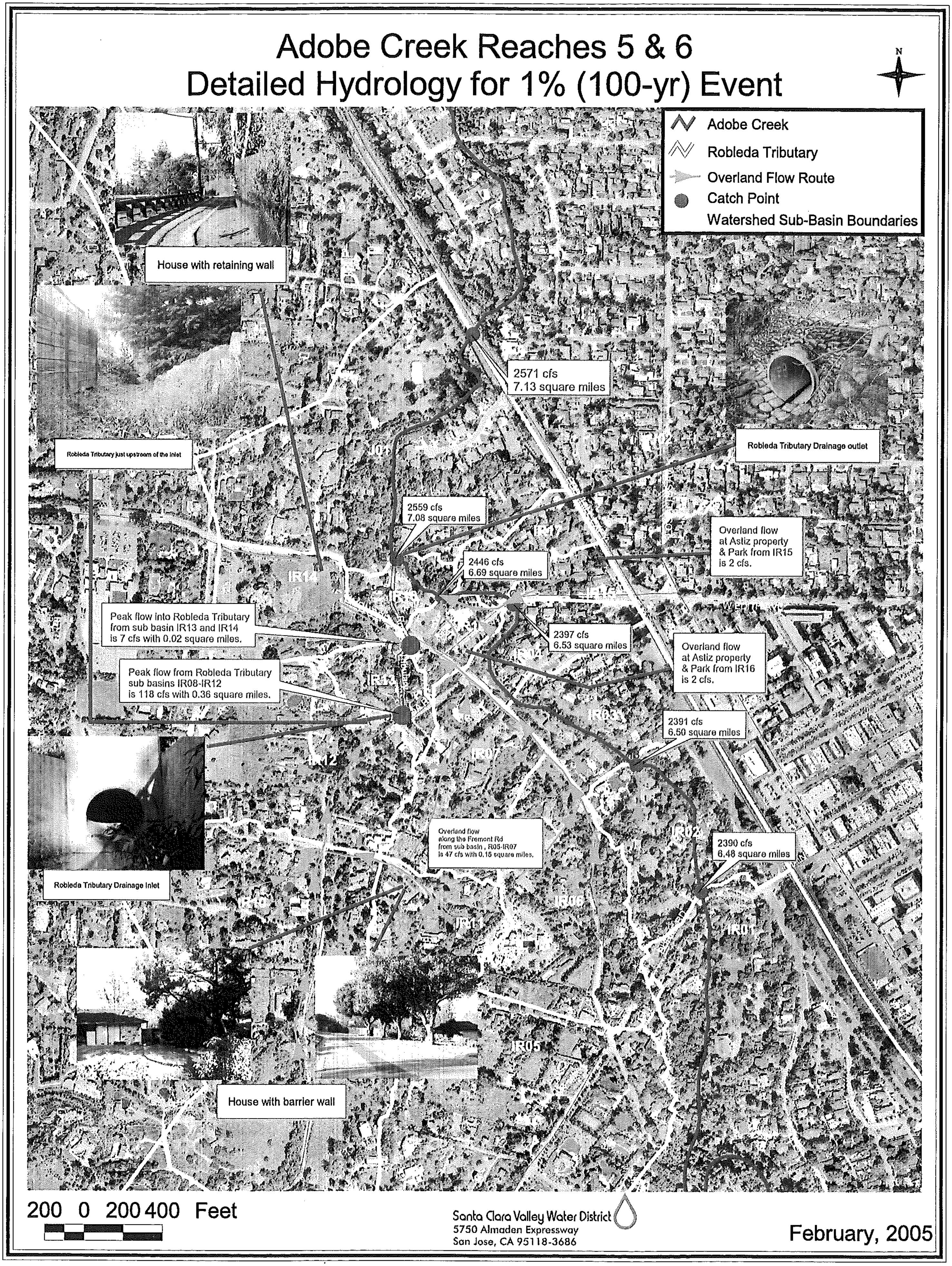


Figure 4. Adobe Creek Reaches 5 & 6 Detailed Hydrology for 1% (100-yr) Event

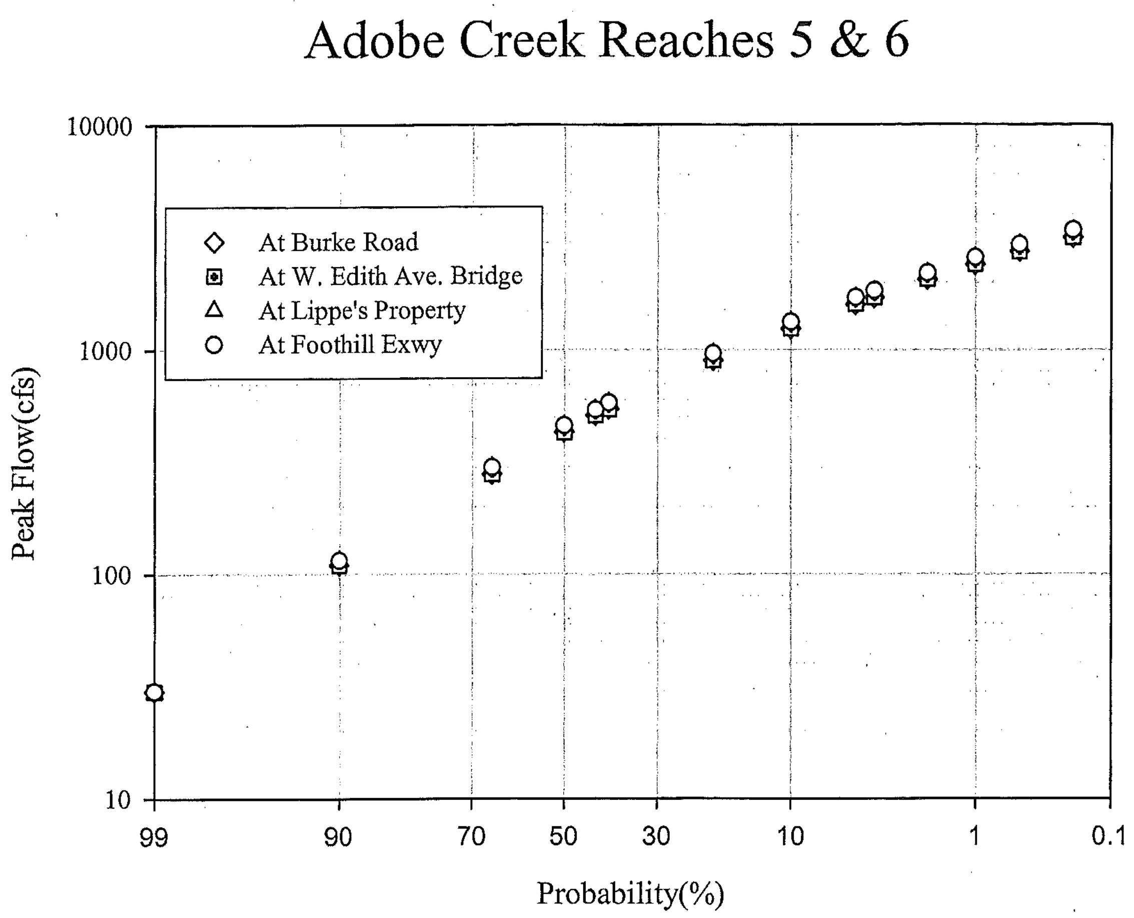


Figure 5. Flood Frequency at Various Locations

APPENDIX A

HYDROLOGY PROCEDURES

For more detailed information on the hydrology procedures, please refer to the District's *Hydrology Procedure*, July 2004.

1. DESIGN STORM PRECIPITATION

The design storm precipitation has been obtained and used to estimate the flood runoff magnitude using an appropriate rainfall-runoff model. For this study, precipitation was obtained by data transpositioned from gaged locations to ungaged site and checked by using the global regional equation for reasonableness. The two methods are described below:

1.1 Global Regional Equations From the isohyetal map of mean annual precipitation (MAP), locate the specific location of the site and determine the MAP. Some interpolation may be required to obtain the MAP. The MAP for the study area ranges from 25 inches to 19 inches as shown in Figure A1 . For this study, the averaged MAP is about 21 inches.

Given the mean annual precipitation for the ungaged site, the precipitation intensity is calculated as:

$$i_{T,D} = \frac{a_1 T^{a_2} \exp[SD_e^{\ 2}/2]}{D^{a_3}}$$

where $i_{T,D}$ is the predicted precipitation intensity (in inches per hour) at return period T (in years as the recurrence intervals) and duration D (in hours); $[a_1, a_2, a_3]$ are model coefficients; and, SD_e is the standard deviation of the model residuals (random term).

The precipitation depth (in inches) $x_{T,D}$ can be simply obtained from the precipitation intensity $i_{T,D}$ by the following relation:

$$x_{T,D} = i_{T,D} . D$$

For short duration rainfall of a 1% event (specifically for durations of 5 minutes, 10 minutes, up to as much as 24 hours), the model parameters $[a_1, a_2, a_3]$ are estimated as follows:

$$\begin{aligned} a_1 &= 0.2675 + 0.01199 \cdot MAP + 0.00002472 \cdot E \\ a_2 &= 0.167033 \\ a_3 &= 0.5853 - 0.004155 \cdot MAP - 0.000001096 \cdot E \end{aligned}$$

where MAP is the mean annual precipitation in inches and E is the elevation of the ungaged site in feet. Note that the parameter a_2 is a constant parameter. One more parameter needed in the $i_{T,D}$ equation is the standard deviation of the model residuals (model error term). The standard deviation for the short duration data is SD_e=0.12003 and for this study, D = 24 hours, E is about 285 feet.

1.2 Data Transposition from Gaged to Ungaged Site For an ungaged site, determine if there is any precipitation gaging station nearby to the ungaged site of interest. If a gaging station exists, obtain the design storm rainfall corresponding to the desired returned period from the historical precipitation-duration-frequency curve for this gaged site. This design storm is assumed to represent the design storm for the ungaged site. If there are significant differences in elevations or topographic features between the ungaged and the gaged site, despite their proximity, their precipitation regimes can be very different. Note that the duration D for short duration rainfall analysis in the $i_{T,D}$ equation is in terms of hours.

2 STREAMFLOW REGRESSION EQUATIONS

Since the early 1970s, the District has utilized regional regression and correlation techniques to estimate design flows at rural and ungaged locations. The regional regression equations are formulas consisting of flow information such as values from gaged stations as dependent variables and measurable watershed characteristics as independent variables. The application of these equations on ungaged locations will result in estimates of flood flow from any watershed for certain selected frequencies (e.g. 1% or 10% design flows). Regional equations usually apply to rural watersheds. The District updated the regional regression model in 2003 to include data through 2000 based on the following criteria:

1. Use the historic data. No stochastic data are included.
2. Use the data from Santa Clara County and Santa Cruz County.
3. Regress the quantiles and not the statistics.

Based on these criteria, the data were reevaluated and the results of the statistical analyses have been utilized in this regression analysis. The outcomes of this study are documented in an addendum report entitled *Development of Regional Regression Equations to Calculate Flood Quantiles in Santa Clara County* (October, 2003). The 2003 regional regression to estimate 24-hr peaks and 1-day volumes for both the 1% and 10% quantiles are as shown below:

FOR PEAKS:

$$Q_{1\%} = 11.22 \times A^{0.954} \times MAP^{1.03}$$
$$Q_{10\%} = 2.985 \times A^{0.988} \times MAP^{1.173}$$

Where:
A = the watershed area in square miles
MAP = the mean annual precipitation in inches

FOR VOLUMES:

$$V_{1\%} = 2.254 \times A^{0.964} \times MAP^{1.187}$$

$$V_{10\%} = 0.895 \times A^{0.933} \times MAP^{1.244}$$

Where:
A = the watershed area in square miles
MAP = the mean annual precipitation in inches

Note: The 24-hr volume is equal to the 1-day volume multiplied by the approximate value of 1.15.

3. LOSS RATES

The loss rates for the rural and pervious parts of sub-watersheds are calculated using the Soil Conservation Service (SCS) Curve Number (CN) method. For all impervious areas, loss rates are assumed to be minimal. The hydrologic soil type, the Antecedent Moisture Condition (AMC) and the ground cover are defined for each sub-watershed and are used to determine the respective Initial Abstraction (Ia) and the CN values for an average watershed condition, AMC 2.

The Antecedent Moisture Conditions are further calibrated for various watershed conditions and various frequencies of occurrences using the flood volumes obtained from the regression model and those obtained from the rainfall-runoff model. Based on this information, the Ia and CN values for the updated AMC are calculated and input into the HEC-1 model. A map of hydrologic soil types for each watershed is included in the Figure A2. The updated AMC is 2.25 for the 1% event and 1.25 for the 10% event in this study.

For the rural and pervious parts of the watershed, the loss parameters are calculated based on the calibration process with regression results. This process of calibration has made the rainfall-generated flood quantiles more reliable as design floods of known probability of occurrence.

4. CLARK'S SYNTHETIC HYDROGRAPH PARAMETERS (Tc and R)

The unit hydrograph parameters applicable to this study are calculated. Rural, pervious and impervious parts of each sub-watershed are considered separately (refer to Figure A3). The Time of Concentration (Tc) is calculated using Kirby Hathaway's formula:

$$Tc = 0.01377[(L * n)^{0.47} S^{-0.235}]$$

Where:
L=distance of overland flow in feet (ft)
n=Manning's watershed roughness coefficient
S=average slope in ft/ft

The Routing Coefficient (R) is calculated based on an acceptable routing indicator: R/(Tc+R). This indicator directly impacts the peaking characteristics of hydrographs. For rural and pervious sub-sub-watersheds, the indicator is assumed to range between 0.5 and 0.9 based on the calibration process with regression quantiles. For impervious sub-sub watersheds, the indicator is assumed to range between 0.1 and 0.5.

5. URBAN HYDROLOGY

In 1996, an urban hydrology procedure was developed which addressed the impact of urban growth on flood flows, and this procedure is used for this study. This procedure accounts for the effects on runoff due to two major urban changes: increased imperviousness and increased channelization. Increased imperviousness reduces the overland flow travel time and thus increases the volume of flow. Increased channelization addresses the impact of conveyance through gutters and storm drains together with the increased storage capability of these facilities. The effects of channelization are most pronounced in flat terrain with slopes less than 0.003. Imperviousness represents coverage from streets, buildings and other lot coverage. The coverage from streets in urban residential areas ranges from 2% to 25%, while for other land uses the value can be as high as 95% of the total lot area. The concept of "Equivalent Street" is obtained from the land use requirements for ratio of streets as a part of the total urban sub-watershed. Based on this equivalent street concept, the length and width of streets in an urbanized area are defined and, hence, the unit hydrograph parameters are calculated. The following are the formulas used in this study to calculate unit hydrograph parameters as input to HEC-1.

Equivalent street length (all measurements are in ft) is calculated by the equation:

$$L_{st}=A_{st}/W_{st}$$

Where:

L_{st} = the equivalent street length
A_{st} = area of streets, (from land use guidelines)
W_{st} = width of streets (from traffic guidelines)

Overland flow length is calculated by the equations:

$$L_i = (A_i - A_{st})/2L_{st}$$
$$L_p = A_p/2L_{st}$$

Where:

L_i = length of overland flow of impervious area
A_i = impervious area, (from land use guidelines)
L_p = length of overland flow of pervious area
A_p = pervious area

The impervious length of overland flow is given by the equation $L_{imp} = L_i + L_{Cb}$ where L_{Cb} is the length of flow to the first catch basin, and L_i is defined above. The pervious length is given by the equation $L_{perv} = L_p + L_{imp}$.

The Time of Concentration (Tc) is calculated separately for pervious and impervious areas using Kirby-Hathaway's formula as defined in Section A.4. The Time of Concentration for impervious areas is calculated as:

$$Tc_{imp} = Tc_i + Tc_{cb}$$

Where:

Tc_{imp} = Time of Concentration for the impervious area
Tc_{i} = Time of Concentration of overland flow over impervious area
Tc_{cb} = Time of Concentration of flow length to first catch basin

Time of concentration for pervious areas is calculated as:

$$Tc_{perv} = Tc_{p} + Tc_{imp}$$

Where:

Tc_{perv} = Time of Concentration for pervious area
Tc_{p} = Time of Concentration of overland flow over pervious area
Tc_{imp} = Time of Concentration of the impervious length of overland flow

The corresponding times of concentration for sub watersheds are listed in Table B1 and B2 in Appendix B.

6. ROUTING PROCEDURES

Most of the flood waters from urban areas are conveyed to flood control channels via storm drain systems. Figure A4 shows the general unitized (prorated to one square mile of an area) storage-discharge rating curves that may be applied to valley urban areas. The combined pervious and impervious inflow hydrographs for a study area are routed through the storm drains using the modified pulse routing method. The storage-discharge relationship for that area is calculated from a unitized relationship and applied to obtain the outflow from the storm drain system. The storage routing usually consists of two types of boundary conditions: namely, "with barrier" (eg. berms, levees, houses and etc.) and "without barrier" conditions. The "with barrier" conditions can be found in the lower parts of a watershed, where the lay of the land has flat or mild slopes (generally less than 0.02). Here, the urban runoff can only reach the creeks through the storm drain system, without the possibility of overland connections. The "without barrier" condition generally exists at the upper part of a watershed where slopes are steep (generally greater than 0.02), or in areas without flood control improvements. Runoff water from these areas without barriers normally finds its overland course and eventually reaches the creek channels.

Muskingum-Cunge channel routing procedures were applied to most watersheds except sub-watershed IR-03, IR-04, IR-17 and IR-18 where the hydrographs were combined directly.

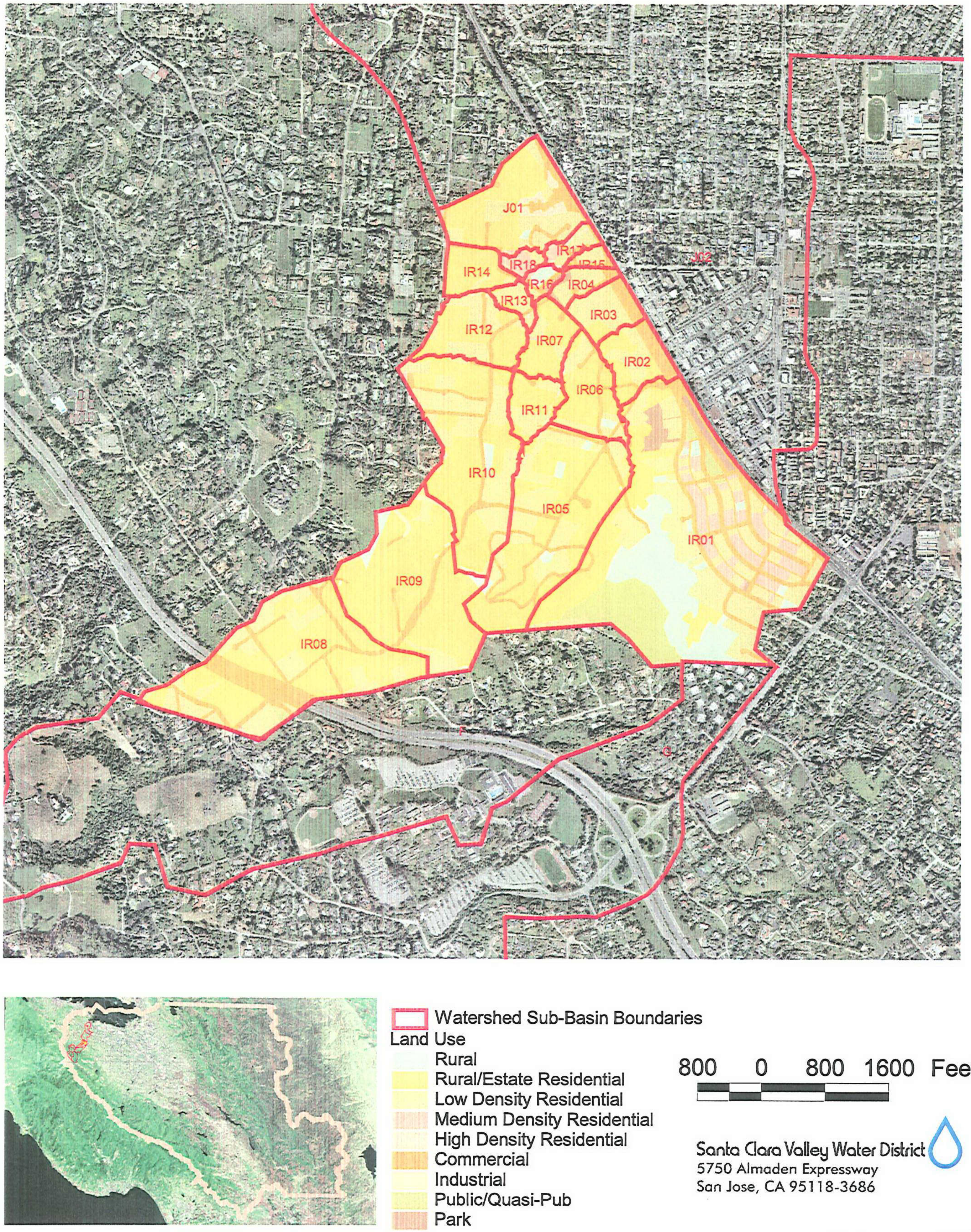
7. DESIGN HYDROGRAPH ESTIMATION FOR RURAL AND URBAN WATERSHEDS

The hydrologic modeling tool used for this study is the HEC-1 Flood Hydrograph Package developed by the U.S. Army Corps of Engineers (1998).

The HEC-1 model has several optional procedures to simulate the various components of the rainfall-runoff process in a watershed. Based on previous modeling studies of rural and urban watersheds around the Santa Clara Valley, the approach adopted by the District in the use of the HEC-1 model for rainfall-runoff modeling is summarized as follows:

- The land use is based on the San Jose General Plan 2020, Santa Clara County Parcel Data (2001), and Santa Clara County Ortho Photos (2001).
- The SCS curve number is calculated based on Hydrologic Soil Groups in Santa Clara County published by the United States Department of Agriculture, Soil Conservation Service.
- The longest rural channel is defined as a sum of the main channel length and overland flow length. The slope for the basin is calculated using the elevation difference divided by the longest rural length.
- Watershed boundaries are delineated using information described in Section 4.1. For rural areas, the boundary is assumed to follow the contour lines. For urban areas, the boundary is assumed to follow the streets, the storm drain system and the natural land contours.
- The SCVWD Maps of Flood Control Facilities and Limits of 1% Flooding are utilized for channel routing information.

Adobe Creek Reaches 5 & 6 Land Use

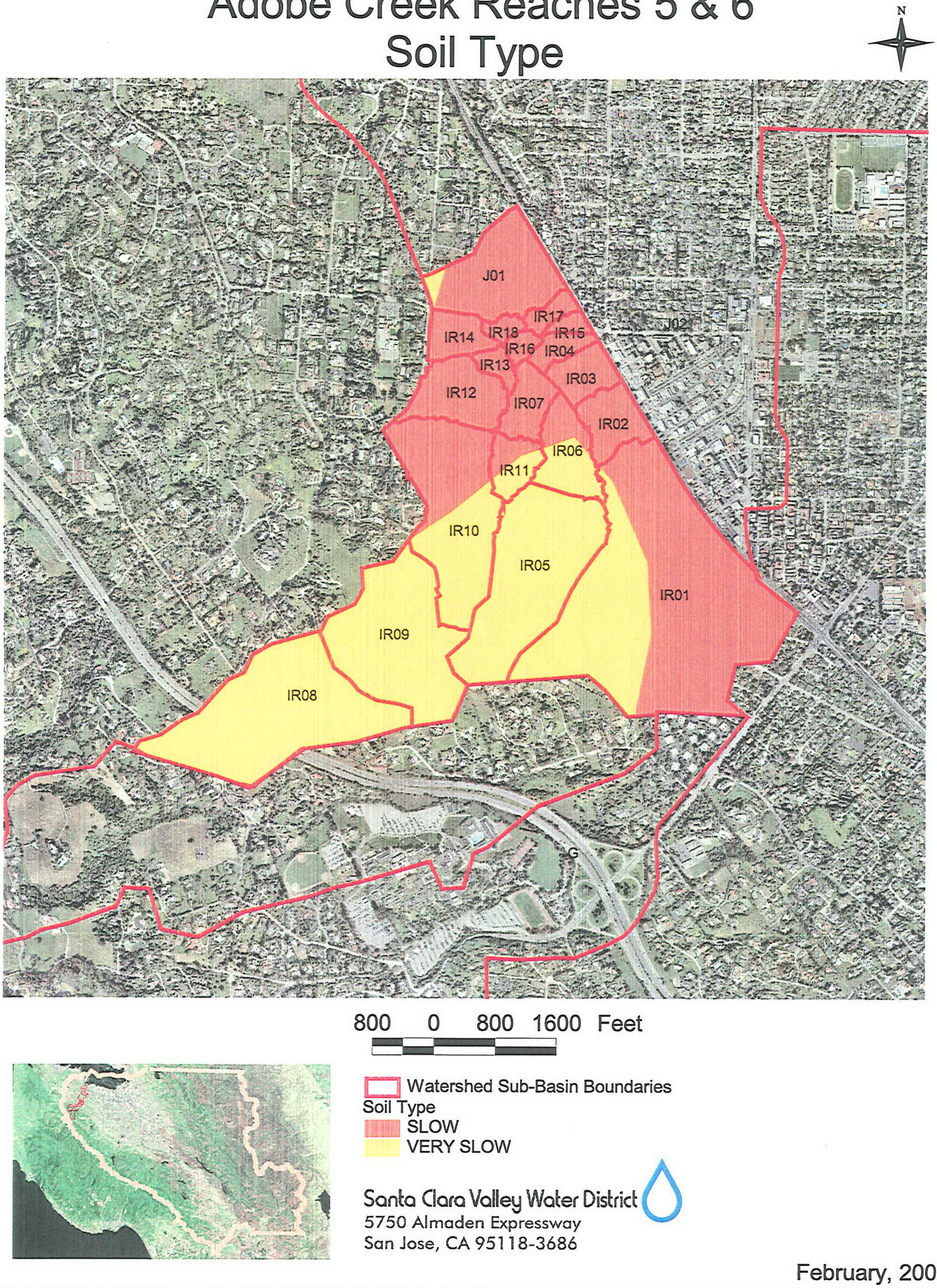


Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

February, 2005

Figure A1. Adobe Creek Reaches 5 & 6 Land Use

Adobe Creek Reaches 5 & 6
Soil Type

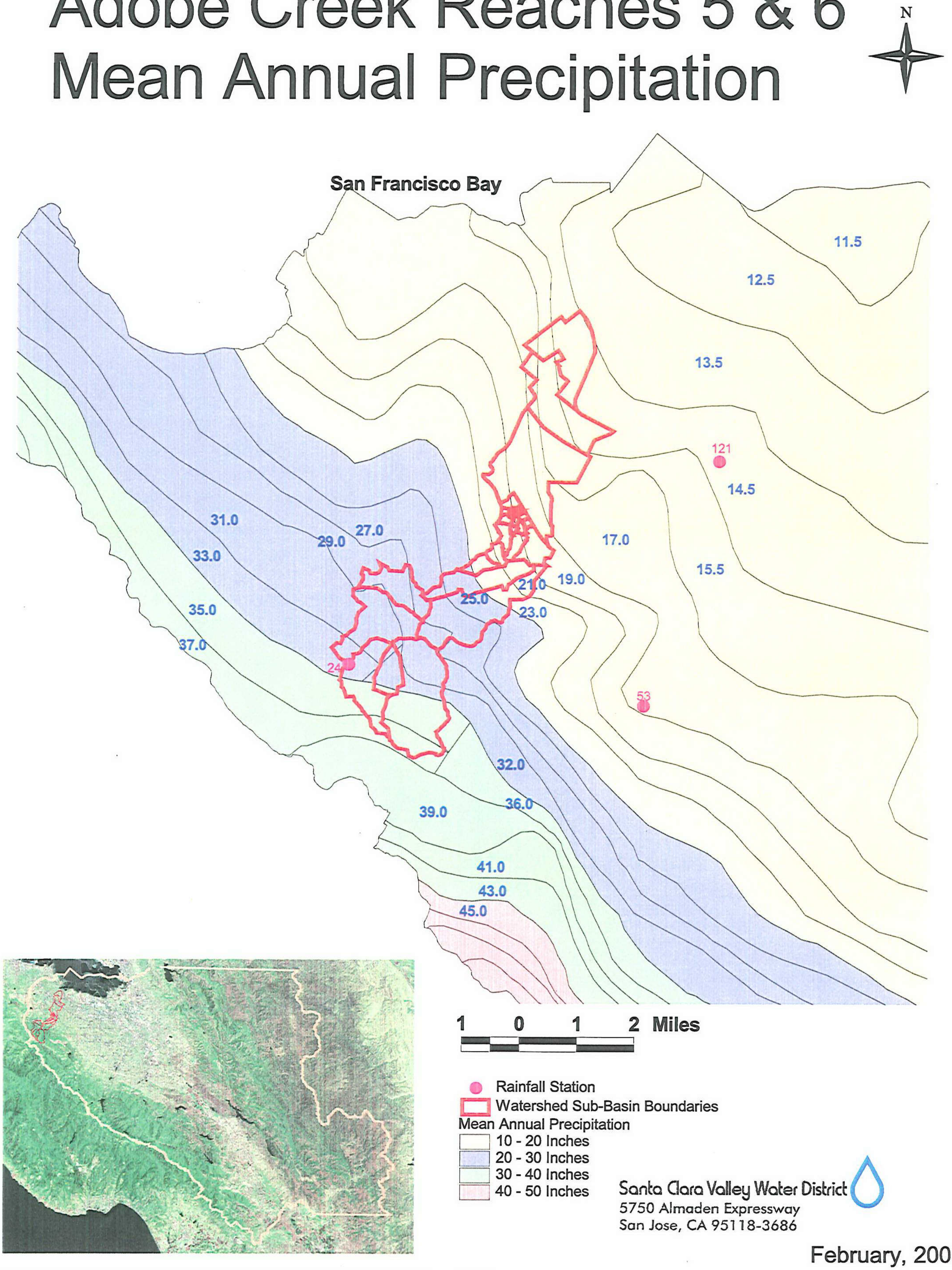


Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

February, 2005

Figure A2. Adobe Creek Reaches 5 & 6 Soil Type

Adobe Creek Reaches 5 & 6 Mean Annual Precipitation



February, 2005

Figure A3. Adobe Creek Reaches 5 & 6 Mean Annual Precipitation

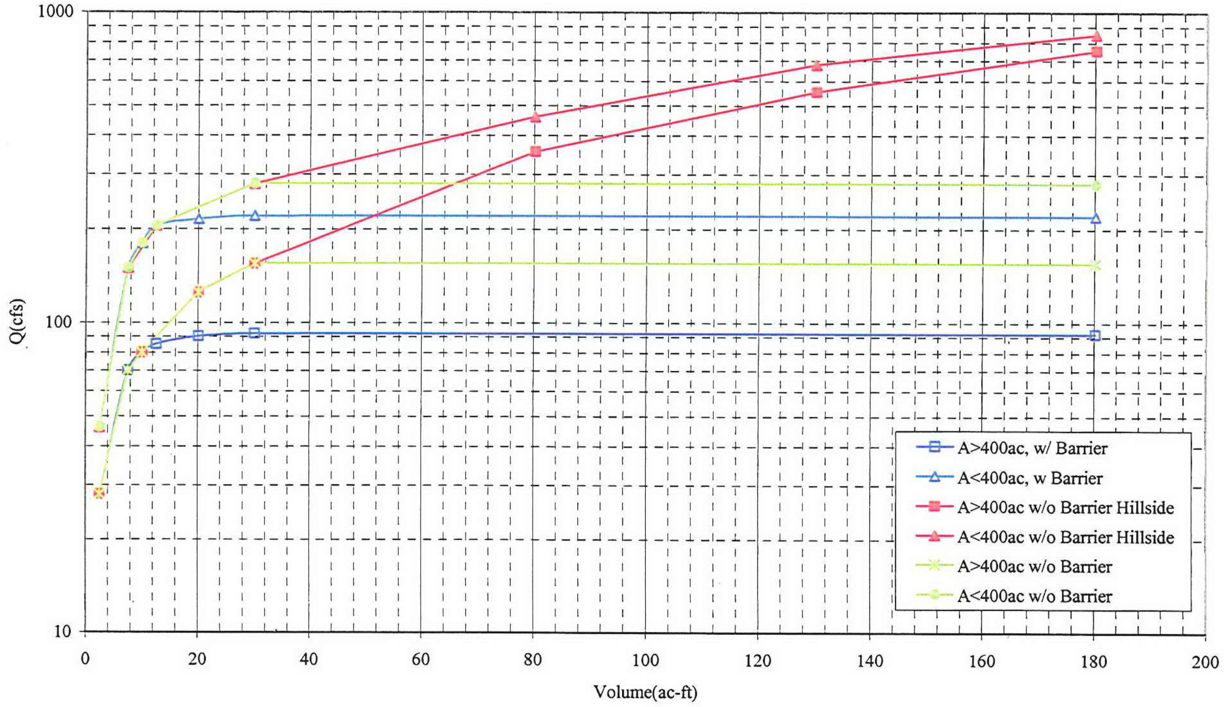


Figure A4. General Unitized Storage-Discharge Rating Curve (prorated to one square mile of an area)

APPENDIX B

Table B1 Adobe (1% HEC-1 Input Data)

	Total Area	Rural Area	Perv Area	Imp. Area	Rural Ia - Calc	Rural CN	Perv Ia - Calc	Perv CN	Imp. Ia - Calc	Imp. CN	Rural Tc	Rural R	Tc_{Perv}	R_{Perv}	Tc_{Imp}	$R_{Imp.}$
BASIN	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
A	0.305	0.305			0.48	81					0.29	0.87				
B	0.769	0.769			0.48	81					0.49	1.48				
C	1.699	1.699			0.59	77					0.56	1.30				
D	0.558	0.238	0.240	0.080	0.68	75	0.42	83	0.04	98	0.50	1.17	0.45	1.06	0.21	0.14
E	0.968	0.795	0.121	0.052	0.48	81	0.32	86	0.04	98	0.50	1.17	0.42	0.99	0.21	0.14
F	0.602	0.074	0.290	0.237	0.65	75	0.40	84	0.04	98	0.30	0.71	0.41	0.96	0.22	0.15
G	1.323	0.349	0.536	0.438	0.59	77	0.42	83	0.04	98	0.31	0.93	0.39	1.18	0.22	0.15
IR01	0.253	0.046	0.093	0.114	0.59	77	0.37	84	0.04	98	0.46	1.38	0.41	1.23	0.23	0.15
IR02	0.022		0.012	0.010			0.42	83	0.04	98			0.51	1.52	0.24	0.16
IR03	0.017		0.009	0.009			0.42	83	0.04	98			0.44	1.31	0.23	0.15
IR04	0.009		0.005	0.005			0.42	83	0.04	98			0.44	1.33	0.23	0.15
IR05	0.102		0.071	0.031			0.30	87	0.04	98			0.41	1.24	0.20	0.14
IR06	0.028		0.020	0.008			0.34	85	0.04	98			0.38	1.14	0.20	0.13
IR07	0.018		0.014	0.005			0.42	83	0.04	98			0.59	1.76	0.23	0.15
IR15	0.004		0.002	0.003			0.42	83	0.04	98			0.40	1.19	0.23	0.15
IR16	0.005	0.002	0.001	0.002	0.68	75	0.42	83	0.04	98	0.22	0.67	0.59	1.78	0.28	0.19
IR17	0.007		0.003	0.004			0.42	83	0.04	98			0.40	1.21	0.24	0.16
IR08	0.123		0.074	0.049			0.30	87	0.04	98			0.37	1.12	0.21	0.14
IR09	0.097		0.073	0.024			0.30	87	0.04	98			0.38	1.14	0.20	0.13
IR10	0.092		0.060	0.032			0.34	85	0.04	98			0.45	1.35	0.22	0.14
IR11	0.016		0.012	0.004			0.34	85	0.04	98			0.40	1.20	0.20	0.13
IR12	0.032		0.022	0.010			0.42	83	0.04	98			0.45	1.34	0.21	0.14
IR13	0.007		0.005	0.002			0.42	83	0.04	98			0.38	1.13	0.20	0.13
IR14	0.015		0.011	0.004			0.42	83	0.04	98			0.45	1.36	0.21	0.14
IR18	0.005		0.002	0.003			0.42	83	0.04	98			0.59	1.76	0.37	0.24
J01	0.056		0.034	0.022			0.42	83	0.04	98			0.48	0.72	0.29	0.20
J02	1.866		0.840	1.026			0.37	84	0.04	98			0.51	0.76	0.33	0.22
K	1.109		0.333	0.776			0.34	85	0.04	98			0.38	0.57	0.23	0.16
L	0.947		0.331	0.616			0.30	87	0.04	98			0.51	0.77	0.26	0.18

1. Total area of the sub-watershed in square miles
2. The rural part of the sub-watershed, area in square miles
3. The pervious part of the sub-watershed, area in square mile
4. The impervious part of the sub-watershed, area in square miles
5. Initial Abstraction (Ia) for rural area
6. Curve Number(CN) for rural area
7. Initial Abstraction (Ia) for pervious area
8. Curve Number(CN) for pervious area
9. Initial Abstraction (Ia) for impervious area
10. Curve Number(CN) for impervious area
11. Times of Concentration(Tc) for rural area
12. Clark's Storage Routing Coefficient R for rural area
13. Times of Concentration(Tc) for pervious area
14. Clark's Storage Routing Coefficient R for pervious area
15. Times of Concentration(Tc) for impervious area
16. Clark's Storage Routing Coefficient R for impervious area

Table B2 Adobe (10% HEC-1 Input Data)

	Total Area	Rural Area	Perv Area	Imp. Area	Rural Ia - Calc	Rural CN	Perv Ia - Calc	Perv CN	Imp. Ia - Calc	Imp. CN	Rural Tc	Rural R	Tc_{Perv}	R_{Perv}	Tc_{Imp}	$R_{Imp.}$
BASIN	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
A	0.305	0.305			1.14	64					0.29	0.87				
B	0.769	0.769			1.14	64					0.49	1.48				
C	1.699	1.699			1.38	59					0.56	1.30				
D	0.558	0.238	0.240	0.080	1.60	56	1.03	66	0.04	98	0.50	1.17	0.45	1.06	0.21	0.14
E	0.968	0.795	0.121	0.052	1.14	64	0.82	71	0.04	98	0.50	1.17	0.42	0.99	0.21	0.14
F	0.602	0.074	0.290	0.237	1.53	57	0.99	67	0.04	98	0.30	0.71	0.41	0.96	0.22	0.15
G	1.323	0.349	0.536	0.438	1.38	59	1.03	66	0.04	98	0.31	0.93	0.39	1.18	0.22	0.15
IR01	0.253	0.046	0.093	0.114	1.38	59	0.93	68	0.04	98	0.46	1.38	0.41	1.23	0.23	0.15
IR02	0.022		0.012	0.010			1.03	66	0.04	98			0.51	1.52	0.24	0.16
IR03	0.017		0.009	0.009			1.03	66	0.04	98			0.44	1.31	0.23	0.15
IR04	0.009		0.005	0.005			1.03	66	0.04	98			0.44	1.33	0.23	0.15
IR05	0.102		0.071	0.031			0.77	72	0.04	98			0.41	1.24	0.20	0.14
IR06	0.028		0.020	0.008			0.87	70	0.04	98			0.38	1.14	0.20	0.13
IR07	0.018		0.014	0.005			1.03	66	0.04	98			0.59	1.76	0.23	0.15
IR15	0.004		0.002	0.003			1.03	66	0.04	98			0.40	1.19	0.23	0.15
IR16	0.005	0.002	0.001	0.002	1.60	56	1.03	66	0.04	98	0.22	0.67	0.59	1.78	0.28	0.19
IR17	0.007		0.003	0.004			1.03	66	0.04	98			0.40	1.21	0.24	0.16
IR08	0.123		0.074	0.049			0.77	72	0.04	98			0.37	1.12	0.21	0.14
IR09	0.097		0.073	0.024			0.77	72	0.04	98			0.38	1.14	0.20	0.13
IR10	0.092		0.060	0.032			0.87	70	0.04	98			0.45	1.35	0.22	0.14
IR11	0.016		0.012	0.004			0.87	70	0.04	98			0.40	1.20	0.20	0.13
IR12	0.032		0.022	0.010			1.03	66	0.04	98			0.45	1.34	0.21	0.14
IR13	0.007		0.005	0.002			1.03	66	0.04	98			0.38	1.13	0.20	0.13
IR14	0.015		0.011	0.004			1.03	66	0.04	98			0.45	1.36	0.21	0.14
IR18	0.005		0.002	0.003			1.03	66	0.04	98			0.59	1.76	0.37	0.24
J01	0.056		0.034	0.022			1.03	66	0.04	98			0.48	0.72	0.29	0.20
J02	1.866		0.840	1.026			0.93	68	0.04	98			0.51	0.76	0.33	0.22
K	1.109		0.333	0.776			0.87	70	0.04	98			0.38	0.57	0.23	0.16
L	0.947		0.331	0.616			0.77	72	0.04	98			0.51	0.77	0.26	0.18

1. Total area of the sub-watershed in square miles
2. The rural part of the sub-watershed, area in square miles
3. The pervious part of the sub-watershed, area in square mile
4. The impervious part of the sub-watershed, area in square miles
5. Initial Abstraction (Ia) for rural area
6. Curve Number(CN) for rural area
7. Initial Abstraction (Ia) for pervious area
8. Curve Number(CN) for pervious area
9. Initial Abstraction (Ia) for impervious area
10. Curve Number(CN) for impervious area
11. Times of Concentration(Tc) for rural area
12. Clark's Storage Routing Coefficient R for rural area
13. Times of Concentration(Tc) for pervious area
14. Clark's Storage Routing Coefficient R for pervious area
15. Times of Concentration(Tc) for impervious area
16. Clark's Storage Routing Coefficient R for impervious area